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Simple connection for square tubular shapes

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SIMPLE CONNECTION FOR SQUARE TUBULAR SHAPES

by

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1. INTRODUCTION

Tubular shaped structural steel has come into use only in recent years. Tubular shapes, mostly round, have been used successfully in the formation of trusses. The problems of connections with round shapes have been investigated and on trusses the usual solution has been various types of welded connections. However, little research has been done on simple beam and column framing connections for square or rectangular structural steel tubing. R. N. White and P. J. Fang[1] have investigated some framing connections, but according to the authors none of the connection studies have the combination of strength and flexibility which is desired for the simple framing connection.

The purpose of this report is to investigate simple framing connections for square tubular shapes and develop a theory for the analysis of these connections. After studying the performance of the framing connection design used for tests in this project, the results were compared with reports of other connection designs previously investigated. With this information a theory for predicting the behavior of these connections was developed.

The framing connection investigated for this report is an 8-1/2 inch plate, 3/4 inch thick, which was fillet welded in a slot in the tubular column. Two sizes of tubes were used, 6 x 6 x 3/16 and 8 x 8 x 1/4; and each tube was tested with light and heavy beams. The tubes and beams were selected so that some comparisons could be made with connections previously tested by R. N. White and P. J. Fang.[1] In all four tests only two-way connections with symmetrical cantilever beams connected to the sides of the square column were studied. The beams were first loaded at their ends and the stresses were held below the yield point of the connection. Next the setup was changed so that the load was applied closer to the column face and the load was increased until failure of the connections occurred. In these tests the only load carried by the column was that transmitted to the column through the connections.

2. DISCUSSION

Because of its shape the square tubular column has similar properties in all directions while the wide flange column has a strong and a weak axis. Although the wide flange shape has a higher moment of inertia in one direction, the square tube has a high moment of inertia in all directions which means it has good bending and torsional resistance. Since the tubular shape also has a high radius of gyration in both directions, it has good strength in compression. Therefore in use as a column section the square tubular shape has more favorable geometric properties than wide flange shapes and could have advantages in some framing conditions.

The problem for square tubular columns has been to find adequate simple framing connections. The connections must be strong enough to carry the beam reactions and yet flexible so only a small bending moment is transmitted to the column wall. If the connection is too stiff, the large bending moment can cause harmful distortions in the tube wall. The connection should be designed such that it does not develop stresses in the column which would cause a great reduction in the column strength. Besides the design requirements the connection must be easy to fabricate and economical.

Because of the tubular shape and ease of fabrication the only practical method of fastening the connection to the tube wall is welding. Since the weld transmits all the stresses to the column, it probably is a critical part of the connection. Using a typical fillet weld the critical stresses in the weld will form in the throat area. Because the weld is an important part of the connection, the plate should be welded over the entire length of the slot. A 3/16 inch weld is the most common and economical since it is placed with one pass of the electrode. It should have sufficient strength for the simple connection tested, but would probably tear under large shear loading conditions.

The tubes used in the testing program were rolled into the square shape and welded with a longitudinal butt weld down the center of one face. The bending and welding of the tube caused residual stresses in the wall. Since the tube was cold worked, the steel has a higher yield in tension but a lower yield in compression. The residual stress distribution for the welded and rolled tube is not known and could cause some significant reduction in the column strength. When the slot is burned in the tube wall, the residual stress distribution is changed since the stress must be zero at the edges of the slot. When the connection plate is welded in the slot, more residual stresses are induced. One question is how detrimental is the connection to the column strength of the tube. The slot burned in the wall may be more beneficial than the additional stresses produced by the weld. It is very difficult to analyze the effect of the simple connection on the column strength without a large number of tests.

When load is applied to the beams framing into the column, the column strength can decrease. As the connection transmits the load to the tube wall, stresses are induced in the wall which may reduce the column strength. With the welded plate the stresses are highly concentrated. Since the connection is welded near the center of the tube wall, the concentration of stresses can cause distortions of the wall. When the load on the beams is increased, high stress concentration form in the wall at the top and bottom of the plate. At these points confined areas of plastic deformation begin while the surrounding elastic areas restrain the wall from large deformations. As the loading increases the plastic areas become larger and the wall distortions increase. When the distortions are clearly visible, the effective area of the tube at the connection has decreased which means a reduction in column strength. The amount of reduction in column strength due to a rotation of the connection would depend on the width to thickness ratio of the column and the ratio of connection length to column size. A complete analysis of this problem is beyond the scope of this report and would require a larger assortment of specimens.

3. SPECIMEN DESCRIBED

For this report tests were performed for simple framing connections to square tubular columns. Two tube sizes of ASTM A36 steel, 6 x 6 x 3/16 and 8 x 8 x 1/4 were used for the tests. Each tube was 4 feet long and had a weld running along its length near the center on one of the walls. The slots for the connections were burned in the tube wall on sides which did not have the weld. This removed one possible problem in analyzing the performance of the tube near the connection. The slots were located 5/16 inch off the centerline at midheight of the tube as shown in Fig. 1. This located the beam at the center of the column face.

The connection plate was cut from 3/8 inch ASTM A36 steel. The same size plate, 8-1/2 inches by 4 inches, was used for all connections. Three 15/16 inch holes were punched in each plate with the line of holes 1-1/4 inches from the plate edge and the holes spaced 3 inches apart as shown in Fig. 1. The slots in the tube were filed and ground until the plate could fit into the slot. The plate was positioned such that the center of the holes were 2 inches from the tube face. The ends of the plate were tack welded to hold it in position as a 3/16 inch weld was placed along the length on both sides of the plate.

Electrical resistance strain gages were placed on the specimen. Three gages were placed on the connection plate on the side opposite the centerline of the tube wall. The gages at the edge were placed as close to the edge as possible. The remaining gages were placed as shown in Fig. 2. There were gages on the inside and outside at the ends of the connection plate in both horizontal and vertical directions. A pair of gages were put on the centerline of the connection wall and unloaded wall. Other instrumentation during testing included extensometers to measure deflection of the loading points on the beam and a rotation gage with a 6 inch gage length on the centerline of the beam above the connection.

Each tube size was tested with heavy and light beams of ASTM A36 steel. The 6 x 6 x 3/16 tube was first tested with 10B15 beams. The 12W31 beams were used for the heavy beams with the 6 x 6 x 3/16 tube and as light beams for the 8 x 8 x 1/4 tube. The heavy beams for the 8 x 8 x 1/4 tube were 18W55. All the beams had three holes punched for the connection to the tube. The holes were centered on the beams 1-3/4 inches from the edge except for the 18W55 beam which had the holes 1-1/2 inches from the edges as shown on Fig. 3. Stiffener plates of 3/8 inch ASTM A36 steel were placed on the beams 6 inches from the holes to give the beam added strength at the loading point for the ultimate load.

The beam was connected to the tube by 7/8 inch ASTM A325 high strength bolts which were 1-3/4 inches long. The beams were aligned perpendicular to the column face and the bolts were tightened by the turn of the nut method. After the bolts were snug, a quarter turn was added to the nuts. This was sufficient to hold the beams securely while the specimen was loaded.

4. TEST PROCEDURES

4.1 Tensile Coupon Tests

Six coupon specimens were made for the tensile coupon tests. Two coupons of ASTM A36 steel were made from each of the following: 6 x 6 x 3/16 tube, 8 x 8 x 1/4 tube, and 3/8 inch plate material. The four coupons from the tubes curved considerably indicating high residual stresses in the tubular structural steel. The coupons were the standard shape as specified by ASTM A370.

The tests were performed on the Tinius Olsen 120^k machine. The automatic recorder was used to graph the strain from the extensometer with an 8 inch gage length and the machine load. The machine was operated with a crosshead speed of 0.025 in./min. and after the steel was stressed beyond the elastic range, the machine was stopped for five minute intervals to obtain static yield. When the strain had reached 0.03 in./min., the extensometer was removed and the machine speed was increased to 0.5 in./min. This speed was constant until failure of the tensile coupon occurred.

4.2 Connection Tests

For the first part of the connection testing the specimen was placed in the Baldwin 300^k machine. An 8 inch spreader beam approximately 6 feet long was used so the load could be applied at the ends of the beam. Mechanical jacks were placed at the ends of the spreader beam and dynamometers were positioned on the jacks as shown in Figs. 4 and 6. With this setup the load on the beams was applied approximately 28 inches from the face of the tube wall. The readings from the dynamometers were used to check for symmetrical loading conditions. The actual load reading was taken from the machine load applied to the column by the spherical head. The specimen was inverted compared to the usual building conditions so that the upper part of the column against the spherical head would be the base in normal column loading.

The loading of the specimen was done in 200 pound increments. Each load was held while strain gage and other instrumentation readings were taken. When one of the strain readings reached the plastic range the testing was stopped and the specimen was saved for testing of the ultimate load capacity of the connection.

To test the ultimate capacity the specimen was placed in the Riehle 800^k machine. Again the column was loaded by a spherical head but the supporting apparatus for testing in this machine was two 2-foot pedestals with 4 inch rollers on which the beams rested. These rollers were centered 8 inches from the column face as shown in Figs. 5 and 7. With this loading setup the load was increased in 2 kip increments until failure of the connection occurred.

For the ultimate capacity testing, the first two specimens were whitewashed. Because of the cold rolled condition of the column section the whitewash did not aid observations and was eliminated for the remaining specimens. On the first test it was noticed that the outer edges of the beam flanges were bearing against the tube wall at relatively low loads. To allow more freedom of rotation to the connection and remove the effect of the flanges at low loading conditions, the outer edges of the beam flanges of the remaining specimens were cut so that the flange could not bear against the corners of the tubular section.

5. TEST RESULTS

5.1 Tensile Coupon Tests

Since the tubular columns had high residual stresses from cold working, the yield point of the steel was higher than expected. The residual stresses also cause a smooth transition from the elastic to plastic range on the stress-strain curve. A 0.2 percent offset was used to find the static yield point. The yield point for the 6 x 6 x 3/16 tube was 57 ksi and for the 8 x 8 x 1/4 tube it was 49.5 ksi. The 3/8 inch plate used for the connection did not have any noticeable residual stresses and produced a more ideal stress-strain curve. The change from the elastic range to the plastic range was obvious. The static yield point was 30 ksi for this steel.

Using the data from the tensile coupon tests, as shown on Table 1, the strain corresponding to the yield point can be determined. This was used as the upper limit in the elastic testing of the connections.

5.2 Connection Tests

The first part of the test was basically a moment loading of the framing connection. When a strain reached the plastic range the load was removed but the strain gages did not return to their original readings. During the testing the only visible movement or rotation was the slight deflection of the ends of the beams, but the rotation gage on the flange above the connection did show that the connection was rotating, as shown in Fig. 8. After the load was removed the beams returned to their original positions and no signs of damage to the specimen were observed.

The next step in the testing was the ultimate capacity of the connection with a larger shear load as compared to moment under usual loading conditions. For each specimen the rotation of the connection was visually observed after a few increments of loading. With the outer edge of the flange cut, the remaining flange bore against the tube wall after approximately six loading increments. As shown in Fig. 9, the rotation of the connection was only slightly constrained by the flange bearing on the wall since the flange was not bearing on the corners of the tubular section.

Besides the visual observation of the bending of the connection wall due to loading, the unloaded wall was also buckled. The square tube is a rigid structural member and the bending of the connection wall caused a corresponding buckling in the unloaded wall. From the loading condition the wall above the connection was forced in and the wall below the connection was pulled out from the column while the unloaded wall of the tube was buckled out above the connection and buckled in below it.

The ultimate failure of the connection was a tension tear at the weld at the bottom of the connection plate. In all tests cracking sounds were heard at high loads before the cracks at the weld were visible. Figure 7 is a picture of a specimen in the machine after failure which illustrates the obvious rotation of the connected beams. For the 6 x 6 x 3/16 specimens the failure appeared to be in the column material while in the 8 x 8 x 1/4 specimens the weld material failed. The 6 x 6 x 3/16 specimens failed at machine loads of 28 kips and 47 kips while the 8 x 8 x 1/4 specimens failed at machine loads of 49 and 46 kips.

On the tube wall the strain gages showed an average of inside and outside readings to be tension. This tendency for the average strain to be tension was the effect of the bending of the wall caused by the rotation of the connection. An example of the strains on the wall around the connection is given in Fig. 10.

The strain readings on the connection plate did not show any consistent pattern. During the elastic testing the lower part of the plate was in tension with the center in slight compression and the upper edge was always compression. In the ultimate capacity tests the pattern of the strains were not consistent in the specimens but the lower part of the plate was always in high tension.

6. ANALYSIS OF RESULTS

The strains measured in the 3/8 inch connecting plates were generally lower than strains in the tube wall. The pattern of strains in the connecting plates varied from test to test as the size of beam was changed. In the final test on each specimen the pattern of strains was also affected by the bearing of the flange of the beam against the tube wall near the end of the test. The strain measurements on the connecting plates were not considered to be significant because failure of the connection would not involve failure of the plate for any of the combinations of beams and columns included in this program.

The pattern of strains in the wall of the tube near the connection produced an indication of the behavior of the tube due to bending and shear stresses produced by loading the beams. The pattern of strains for any one specimen was similar for the loading in the moment-rotation test and the ultimate load test. Because of this, only the strain data from the ultimate load test have been plotted for each specimen.

The transverse strain in the tube wall at the top and bottom of the connecting plate resulted in a very consistent pattern. This may be seen by comparison of Figs. 11 and 12. The gages on the outside and inside surfaces at both locations gave relatively large strain readings with the average strain being in tension for the upper side of the plate and approximately zero for the other side. For some tests the average strain was compression for the lower gages.

The strain data for the vertical gages at the same locations was not as consistent as the data from the horizontal gages. The average strain above the beam was tension, but the pattern of strains for the inside and outside gages varied from test to test.

The vertical strains measured on the wall below the connection were more consistent. Generally the strain on the outside wall was tension from the start changing toward compression for higher loads. The strain on the inside wall was compression at the start changing to tension at higher loads. A typical pattern of strains is shown in Fig. 13. The average strain was tension at this location throughout the test. Failure of the connections resulted from tearing of the welds at this location. This would seem to correlate with the high average tensile strains read on the vertical gages.

The strain gages centered on the tube were located 2 inches above the connection plate and served to indicate the extent of the local effect resulting from the loads on the connection. The length of the column which is affected by the connection loading is a function of the column size, the beam size, and the magnitude of the applied load. Figures 14 and 15 show that there is an appreciable bending strain present at the location of these gages. Since the average strain is tension at this location, the effect of the connection is to cause other parts of the cross section to carry the compression load.

There is a tendency for the loaded sides of the column to be in compression at very high loads.

Figure 16 shows that the unloaded sides of the column tend to have a net tension stress also. This indicates that the compressive stress is carried at the four corners of the column near the connection. The unloaded walls exhibited a tendency to deflect outward while the loaded walls deflected inward. Since the residual stress at the corners of the column is apt to be tension, the redistribution of stress in the cross section at the connection does not tend to reduce the column strength even though the bending in the walls is considerable.

If one considers the results of the column tests reported in Ref. 1, it will be noted that the results of these tests are not unlike the results to be expected if only the bare column were loaded. In other words, the connection is not more detrimental to the column strength than is the residual stress pattern in the original section. In fact the fabrication procedure used in making this connection may produce a more favorable distribution of residual stress in the vicinity of the connection. This would help to explain why the connection did not produce lower than normal results in the column tests of Ref. 1. This aspect of the problem deserves further study.

7. SUMMARY AND CONCLUSION

The 8-1/2 inch plate used in these tests seems to be the proper size for connecting the beams involved with the tube sections included in the program. This size of plate is a little large for the 10B15 shape. In testing the ultimate capacity of the connection the shear load did not reach half of the allowable shear for the standard 8-1/2 inch connection which consisted of two 8-1/2 inch angles as given in the AISC Manual. The welded joint does not have the necessary strength to carry a force equal to the capacity of the beams. In tests performed by White and Fang[1] the connection tested was able to sustain much larger shear loads, but it should be noted that their tests were made with a pure shear loading.

Another important characteristic of the simple connection is the flexibility as determined by the moment-rotation characteristics of the connection. In the tests performed the average moment required for a 0.01 radian rotation was 14 kip-inches for the 6 x 6 x 3/16 tubes and 18 kip-inches for the 8 x 8 x 1/4 tubes. As shown in Table 2, these values are lower than any connection tested by White and Fang. Therefore this connection is more flexible than previous framing connections tested for square tubular columns. Greater flexibility is generally considered to be a desirable characteristic for simple connections.

The weld is the critical area in the simple framing connection. The shear capacity of the weld was never reached because the applied moment produced a tearing of the weld at the bottom of the plate. For an adequate connection to be found, a weld size which allows the right combination of strength and flexibility must be selected. The fabrication process of this connection is an important element for this connection. If the slot is not properly prepared, the full capabilities of the weld can not be developed. The most important part of the weld is at the end of the plate which is also the most difficult place to achieve the quality needed in the fabrication. If some fabrication method could be found which would develop a satisfactory weld at the ends where it is needed, the capacity of the connection would be greatly increased. From these tests it appears that a larger weld could satisfactorily be used for these connections, and a smaller clearance in the slot is desirable.

The type of connection tested is adequate for light loads, but should be used with caution where heavy loads are encountered. More tests are required with different size connection plates and varied column and beam combinations before a satisfactory theory for the connection behavior can be formed. With further studies this type of connection may prove to be a good solution for simple framing connections to square tubular columns.

TABLE 1 TENSILE COUPON TEST RESULTS

Coupon No.	Source of Coupon	Static Yield Stress (psi)	Tensile Strength (psi)	Reduction of Area[a] (%)	Elongation[b] (%)
A-1	6 x 6 x 3/16 tube	57,100	71,600	41.9	17.6
A-2	6 x 6 x 3/16 tube	56,500	72,300	45.7	23.8
B-1	8 x 8 x 1/4 tube	50,200	65,700	47.2	21.6
B-2	8 x 8 x 1/4 tube	48,900	63,300	46.8	24.8
C-1	3/8 inch plate	29,600	53,500	53.4	30.5
C-2	3/8 inch plate	30,200	53,600	52.5	29.7

[a]Reduction of area at location of necking

[b]Elongation over the 8 inch gage length

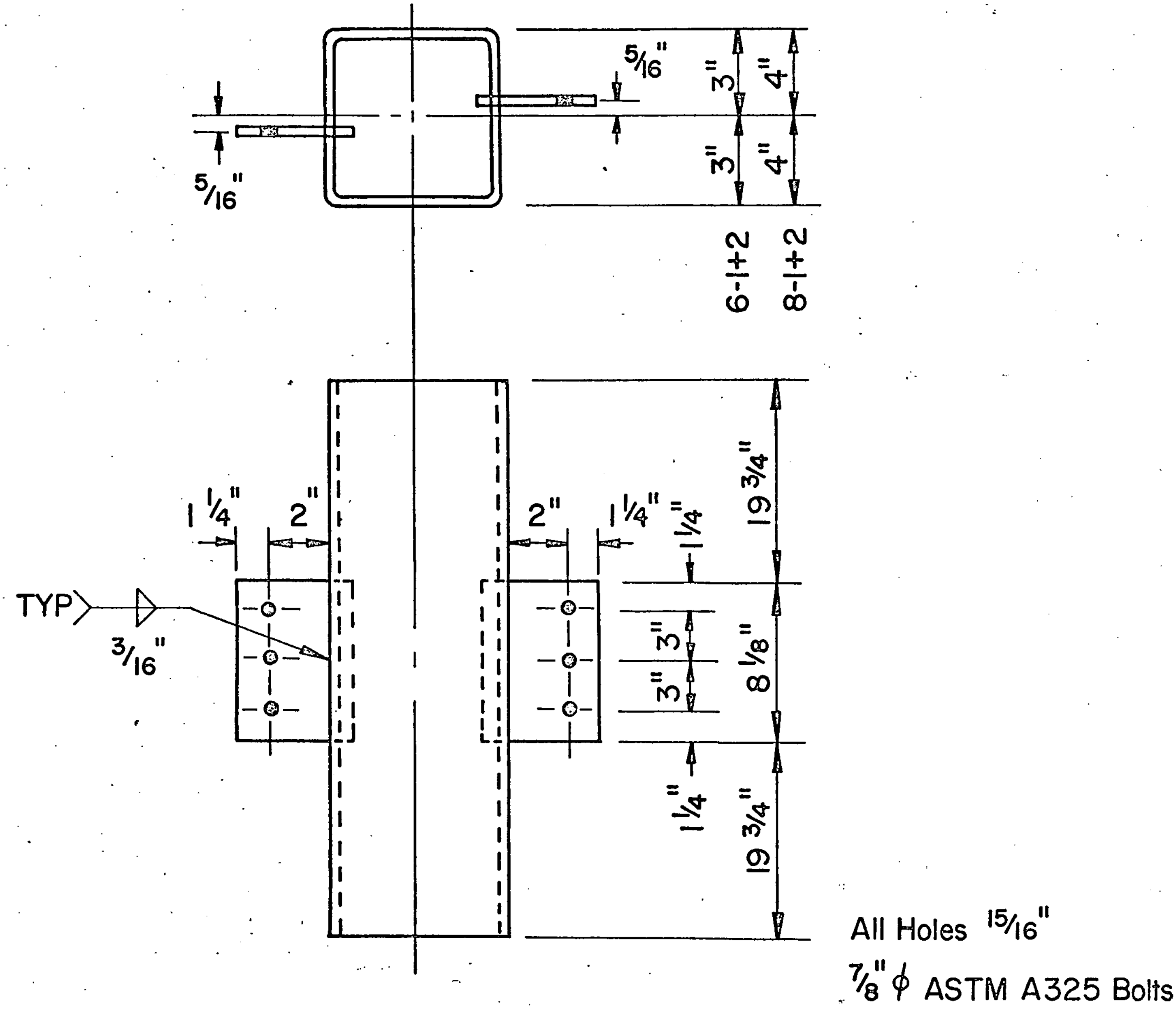
TABLE 2 CONNECTION TEST RESULTS

Specimen	Tube Size[a]	Beam Size	Moir[b] (kip-inches)	Shear Test Load (kips)
D-1[c]	6 x 6 x 3/16	12W27	140	--
E-1[c]	6 x 6 x 3/16	12W27	135	--
6-1	6 x 6 x 3/16	10B15	13	28.0
6-2	6 x 6 x 3/16	12W31	14	27.3
A-5[c]	8 x 8 x 1/4	12W31	22	108
8-1	8 x 8 x 1/4	12W31	19	48.0
8-2	8 x 8 x 1/4	18W55	18	46.0

[a]All connection plates are 8-1/2 inch (3/8 inch ASTM A36) with the 7/8 inch H. S. bolts.

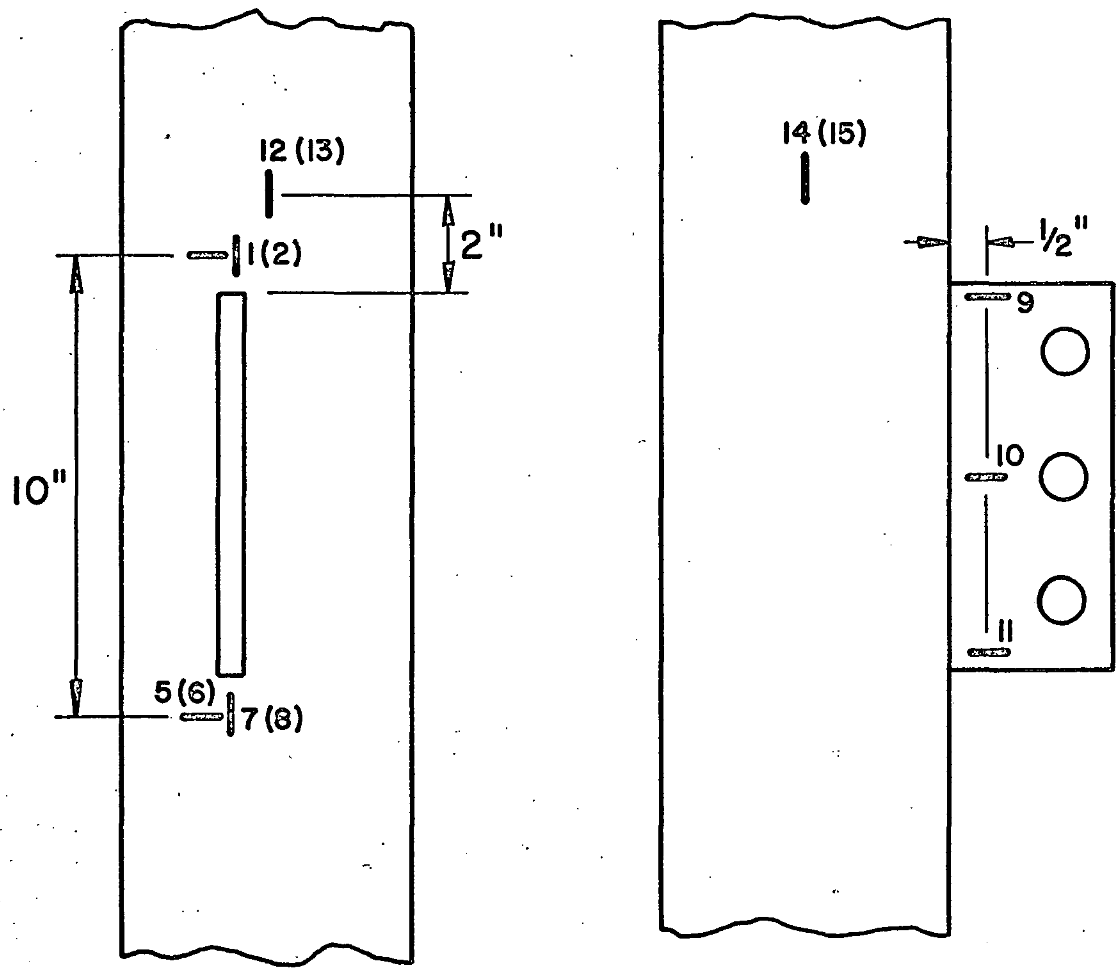
[b]Connection moment for rotation of 0.01 radians.

[c]Specimens from White and Fang tests.



SPEC.	TUBE	PLATE	BEAMS
6x1	6x6 3/16	4 3/8x8 1/8	10B15
6x2	6x6 3/16	4 3/8x8 1/8	12WF3
8x1	8x8 3/16	4 3/8x8 1/8	12WF3
8x2	8x8 3/16	4 3/8x8 1/8	18WF55

Fig. 1 SIMPLE FRAMING CONNECTION DESIGN



Gages in parentheses are
on inside of tube wall

Fig. 2 LOCATION OF ELECTRICAL STRAIN GAGES

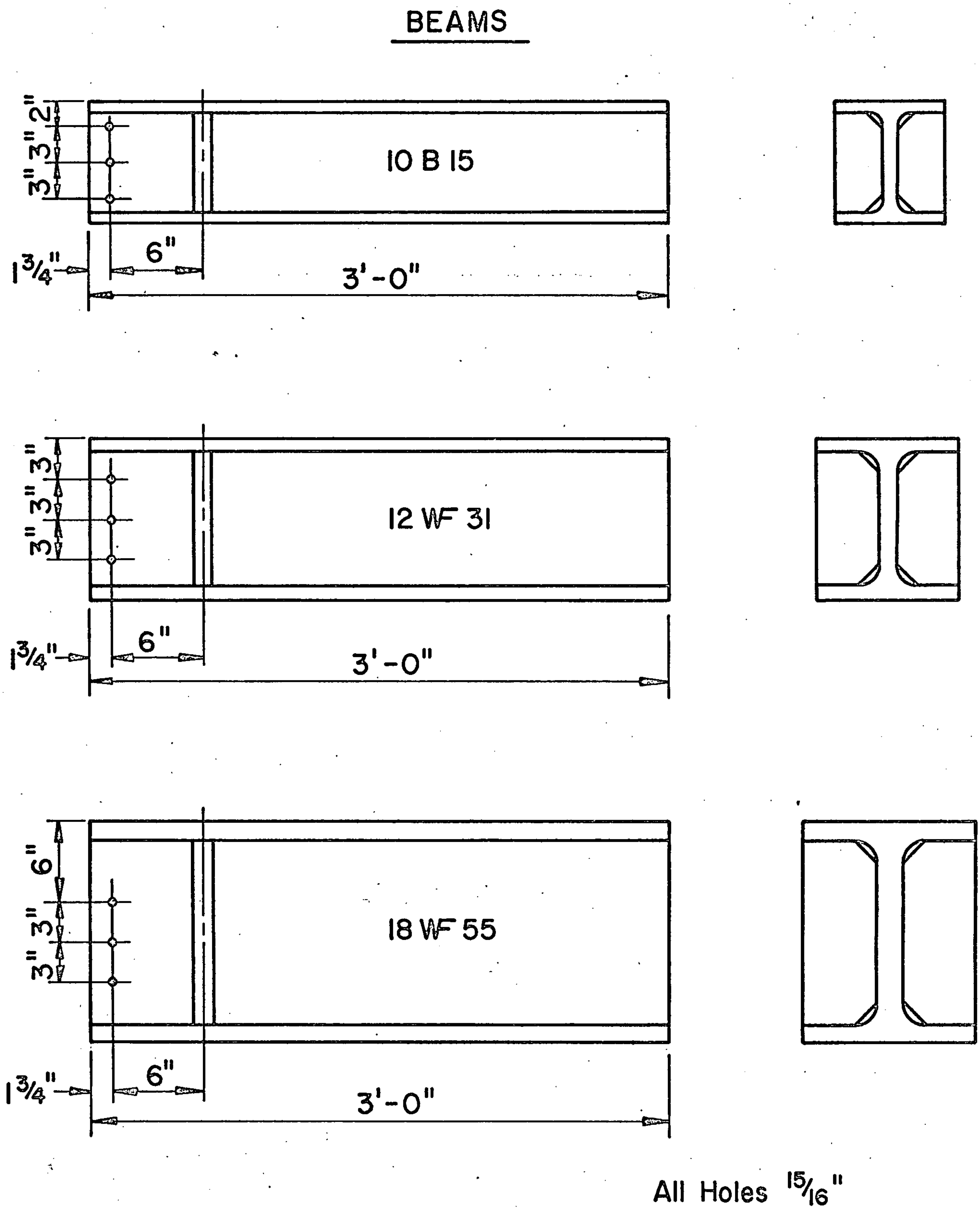


Fig. 3 CONNECTING BEAMS

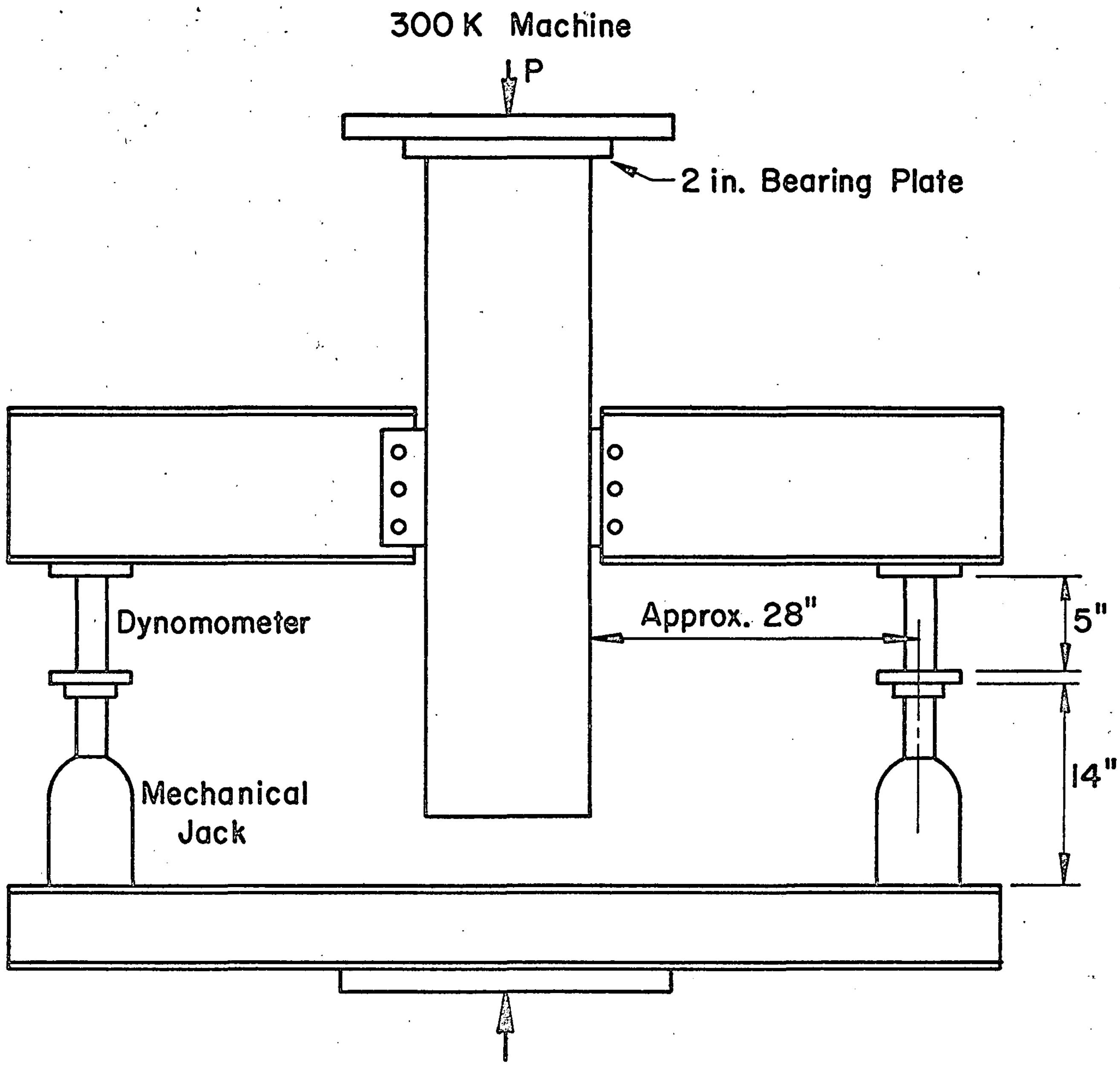


Fig. 4 SKETCH OF ELASTIC LOADING TEST SETUP

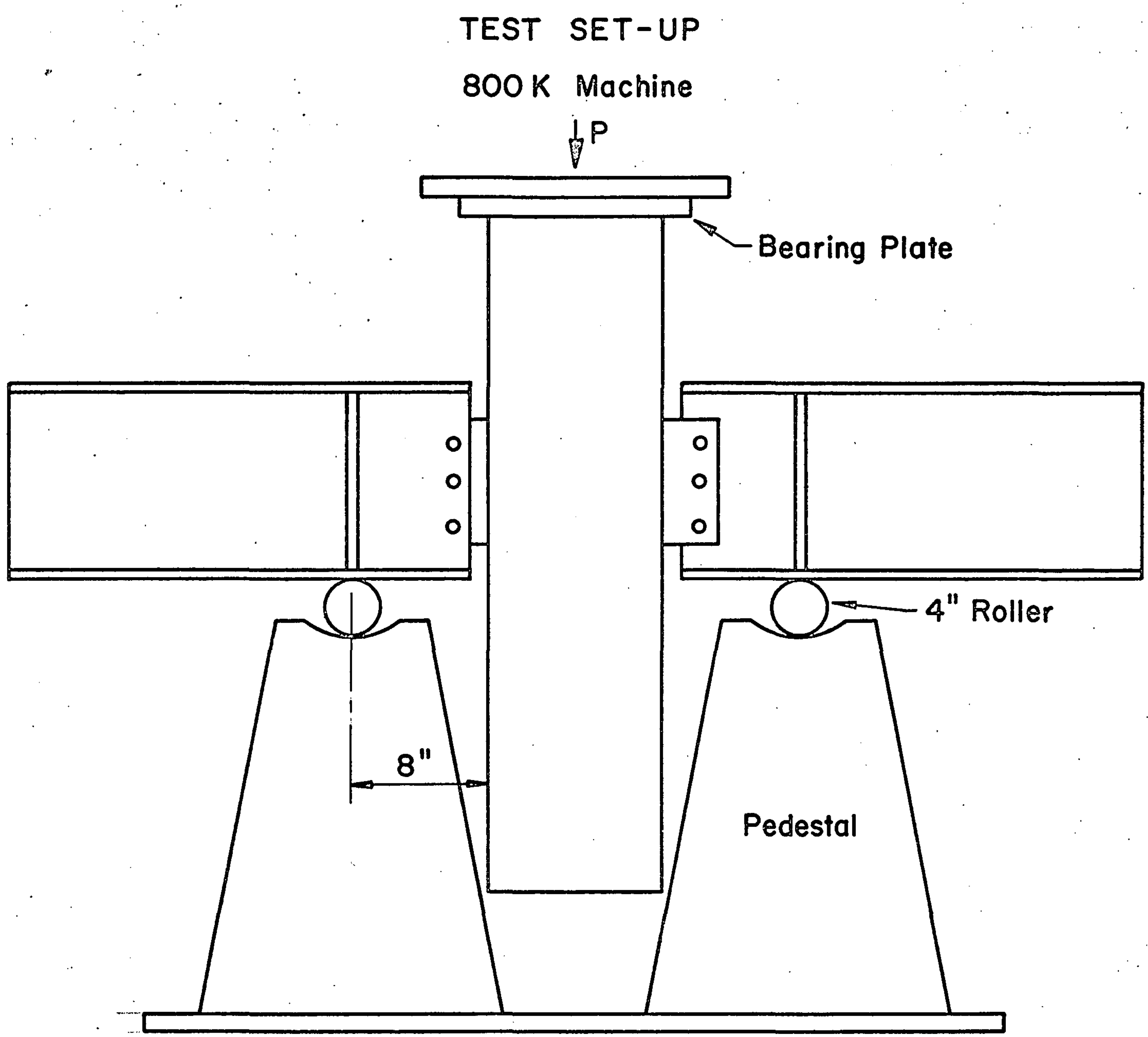


Fig. 5 SKETCH OF ULTIMATE CAPACITY TEST SETUP

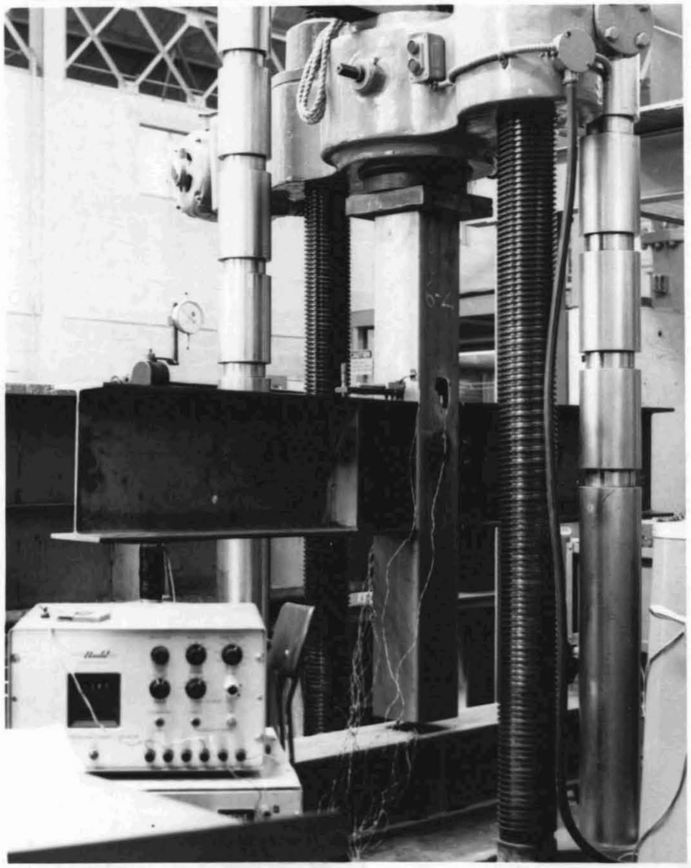


Fig. 6

TEST SETUP FOR ELASTIC LOADING

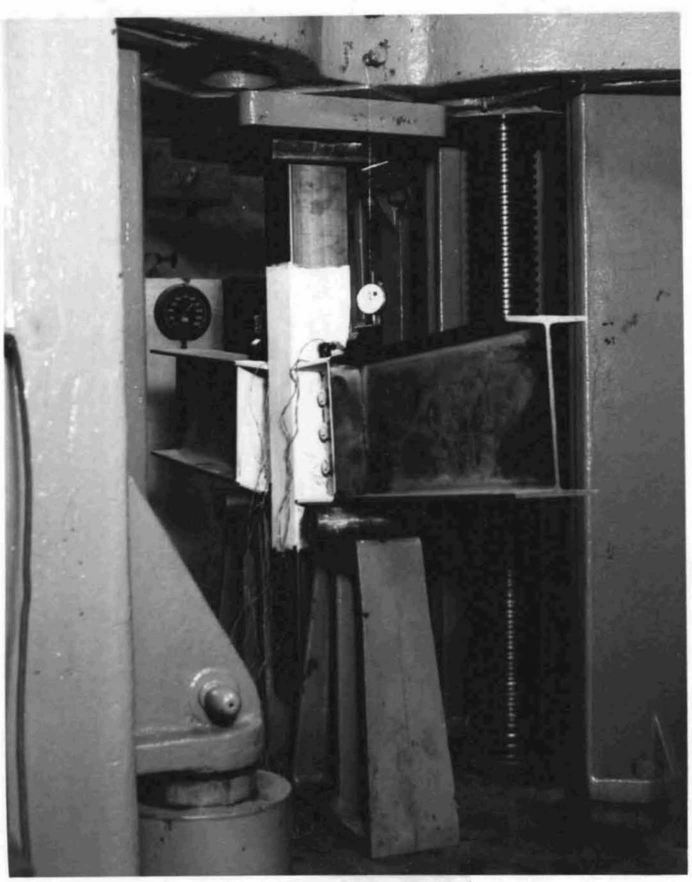


Fig. 7

TEST SETUP FOR ULTIMATE CAPACITY TESTING

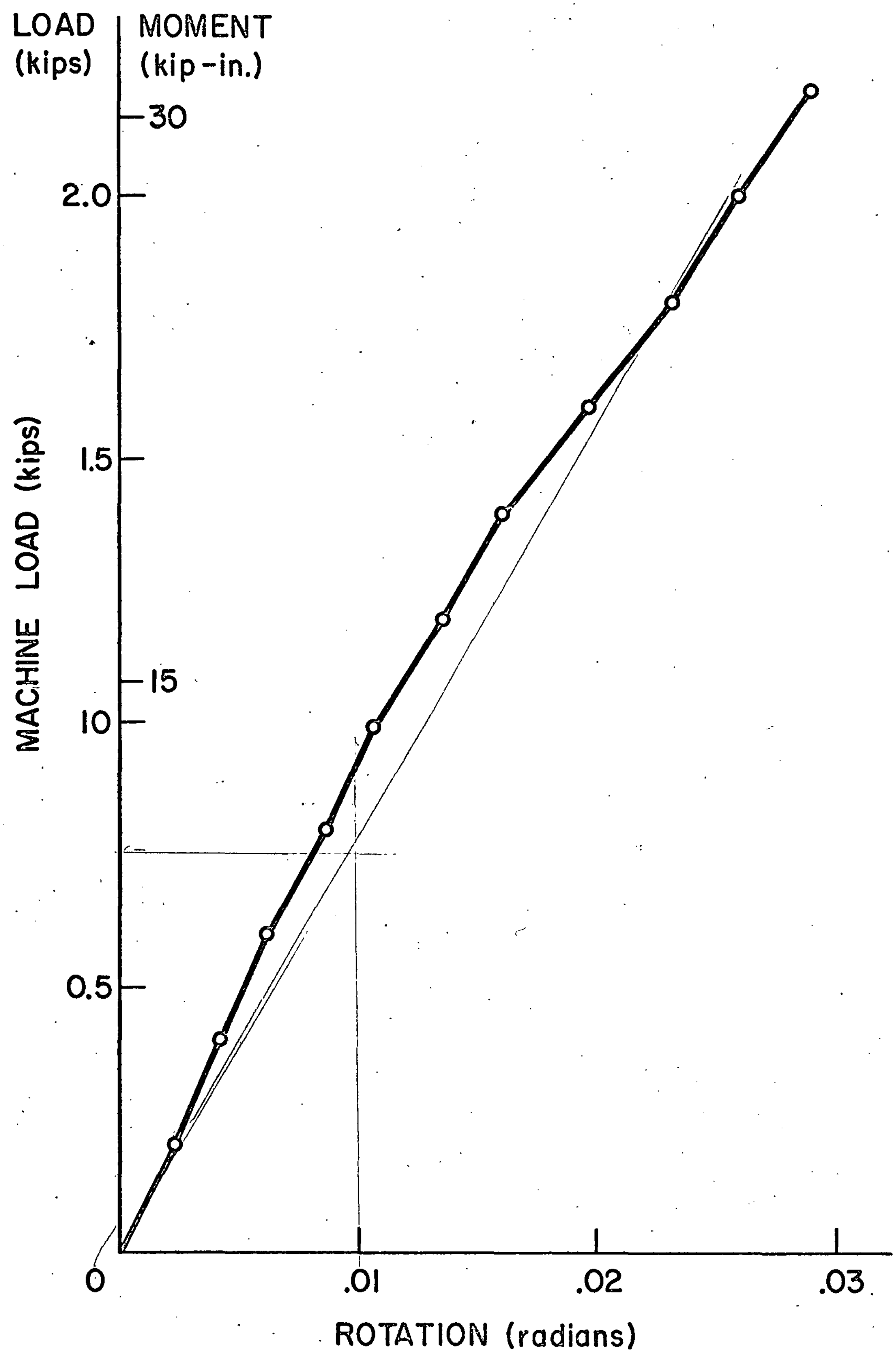


Fig. 8 MOMENT VERSUS ROTATION FOR ELASTIC LOADING OF 6-2

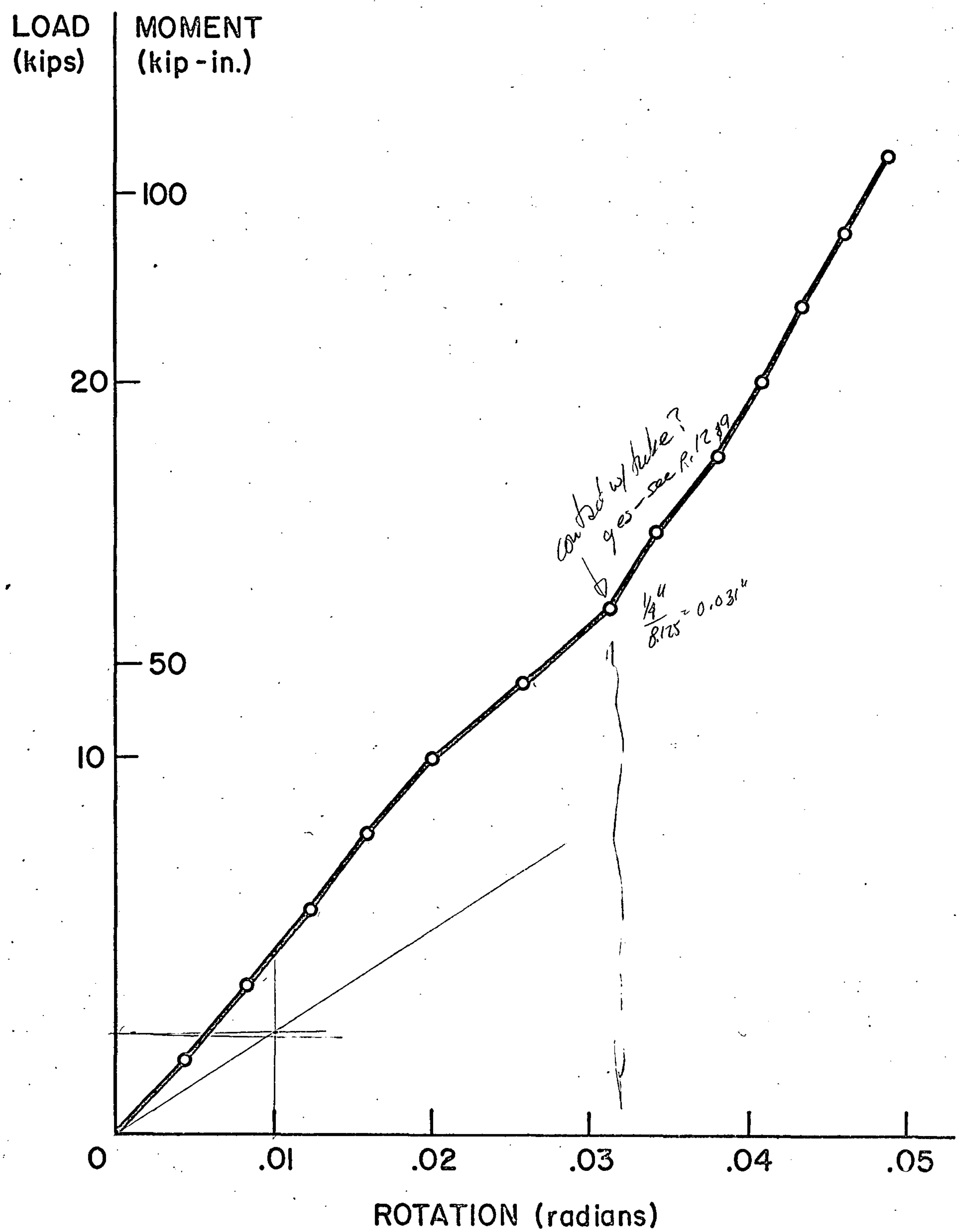


Fig. 9 MOMENT VERSUS ROTATION FOR ULTIMATE CAPACITY TEST OF 8-1

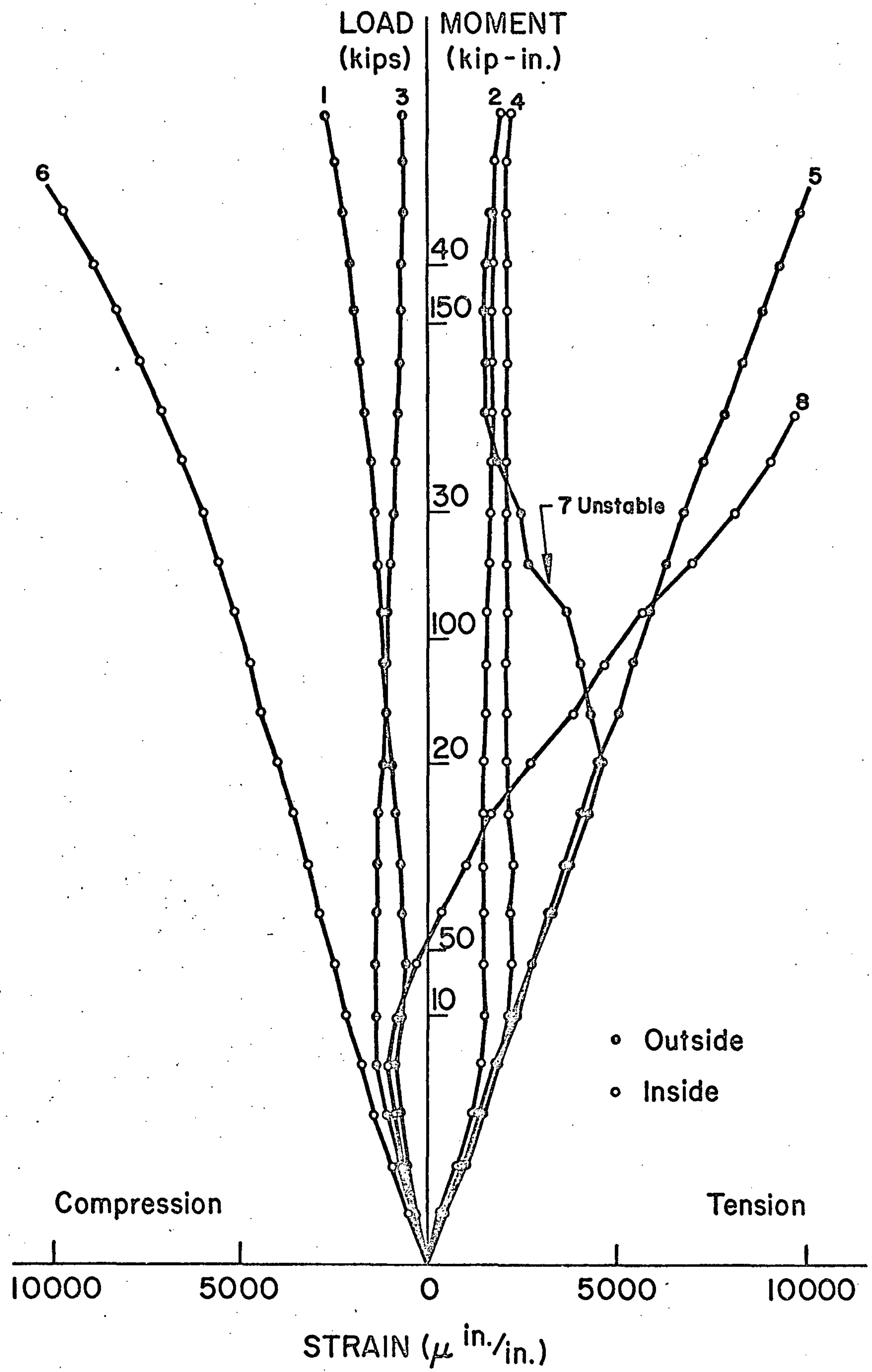


Fig. 10 TUBE WALL STRAINS AROUND CONNECTION OF 6-2

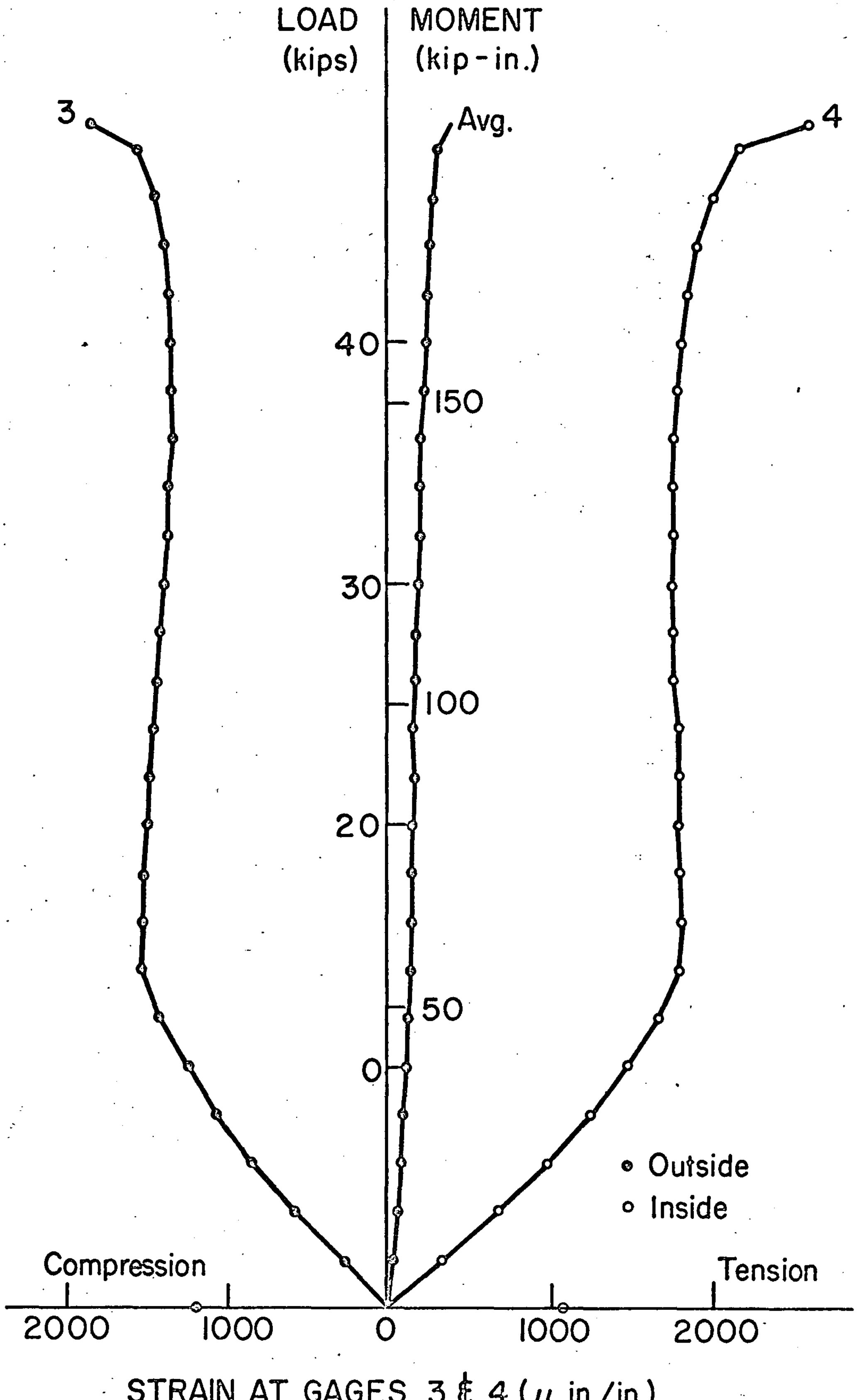


Fig. 11 HORIZONTAL WALL STRAINS NEAR TOP OF CONNECTION OF 8-1

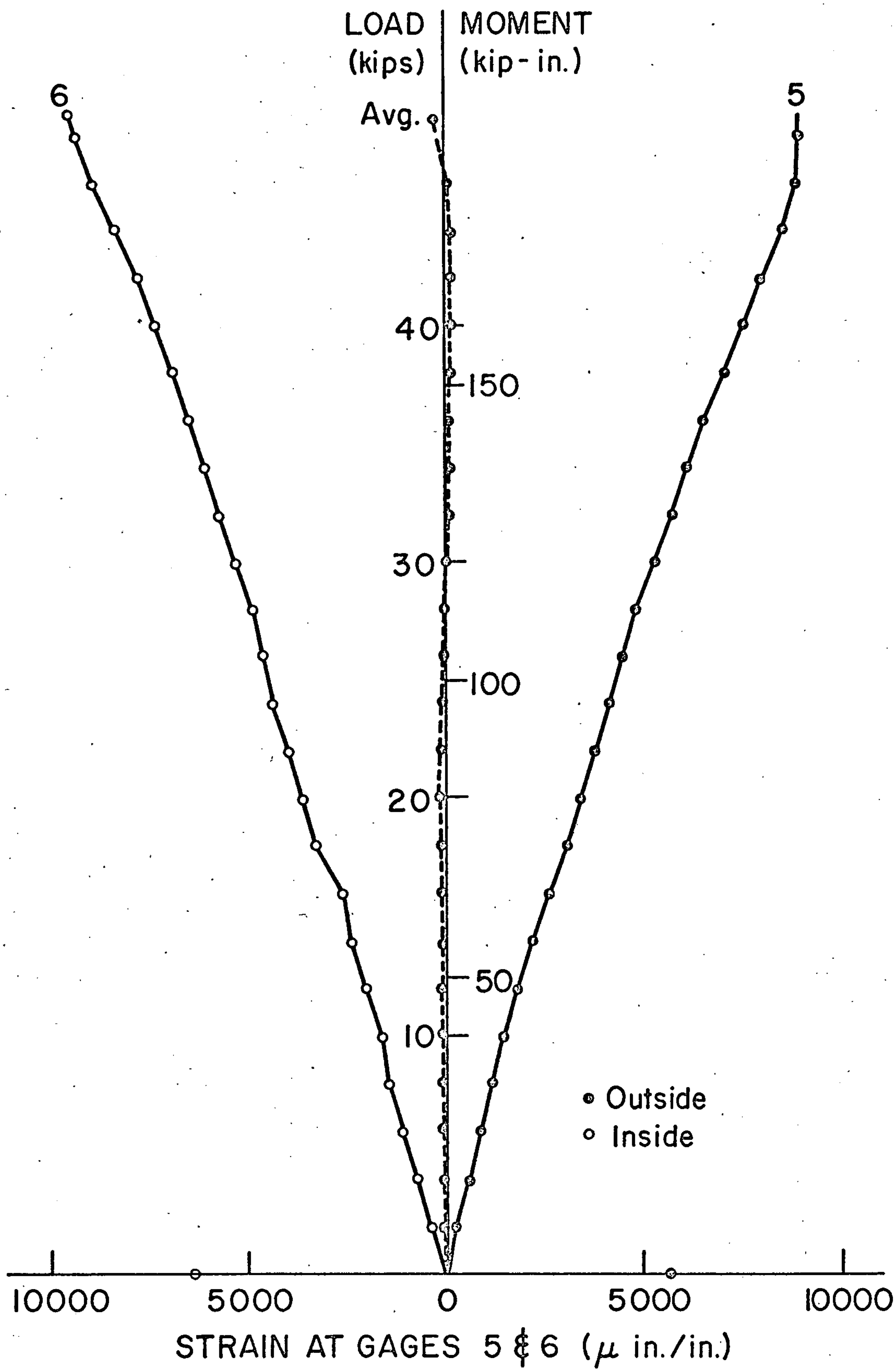


Fig. 12 HORIZONTAL WALL STRAIN NEAR BOTTOM OF CONNECTION OF 8-1

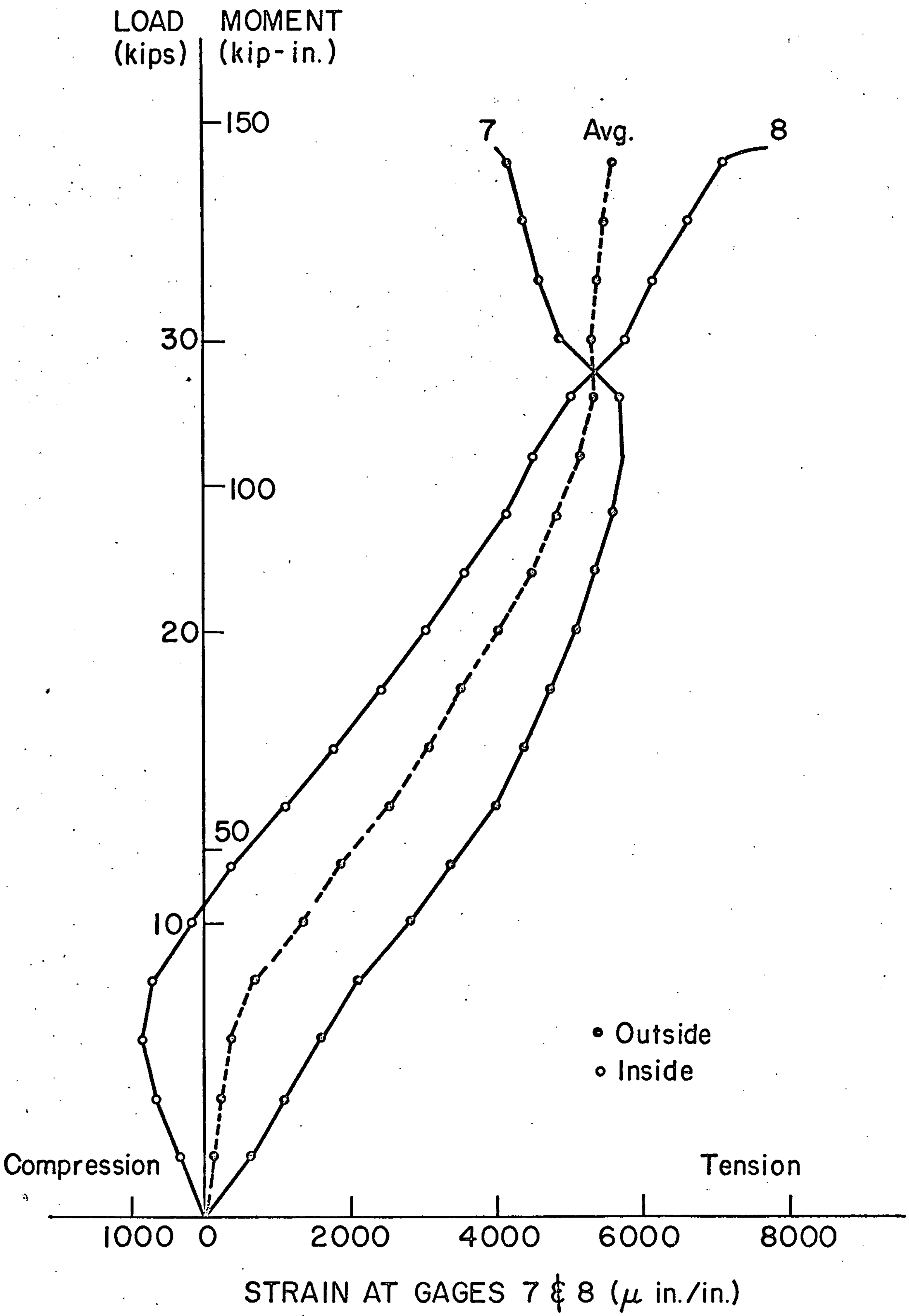


Fig. 13 VERTICAL WALL STRAIN NEAR BOTTOM OF CONNECTION OF 8-2

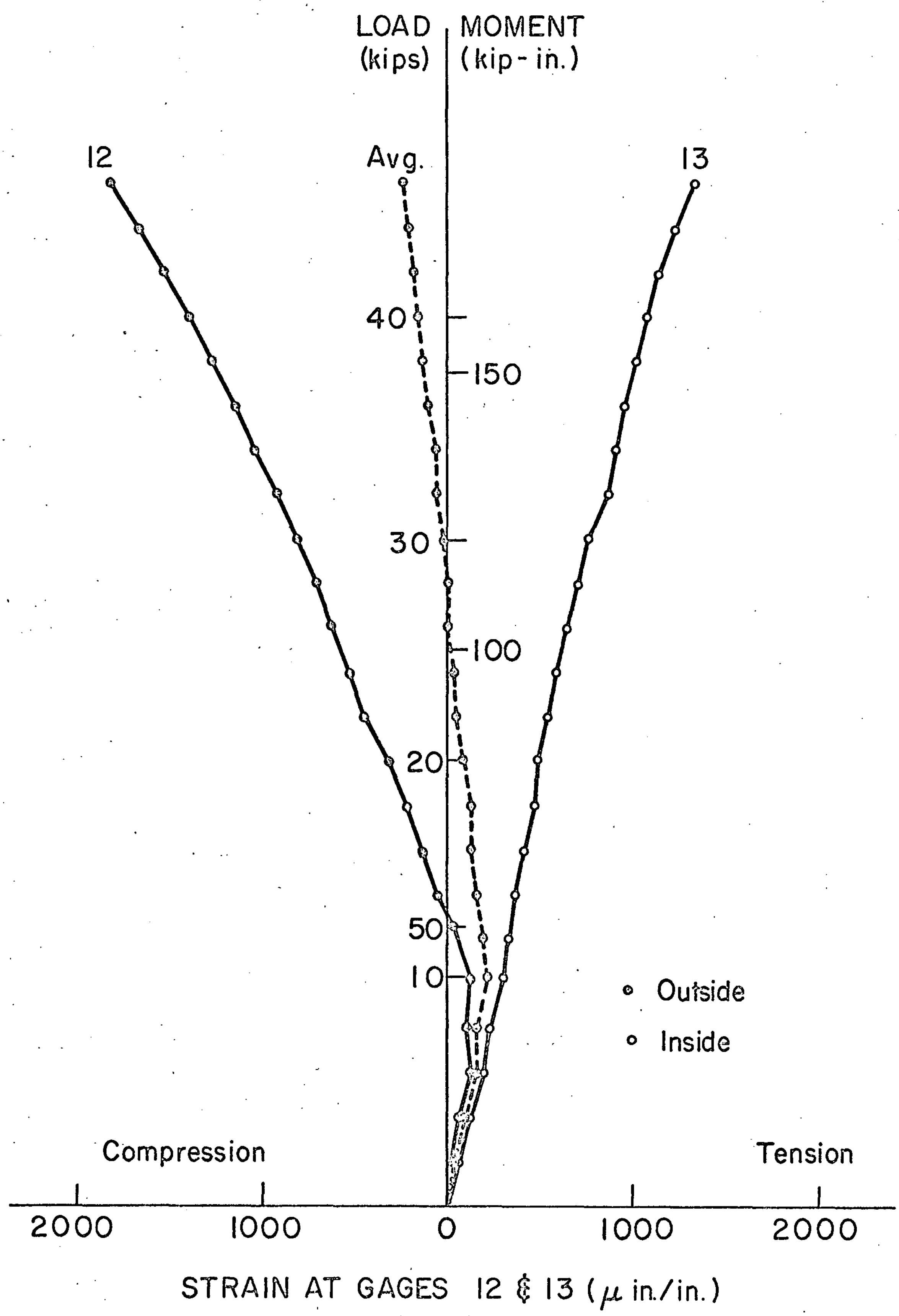


Fig. 14 VERTICAL WALL STRAINS OF CONNECTION WALL ABOVE CONNECTION OF 6-2

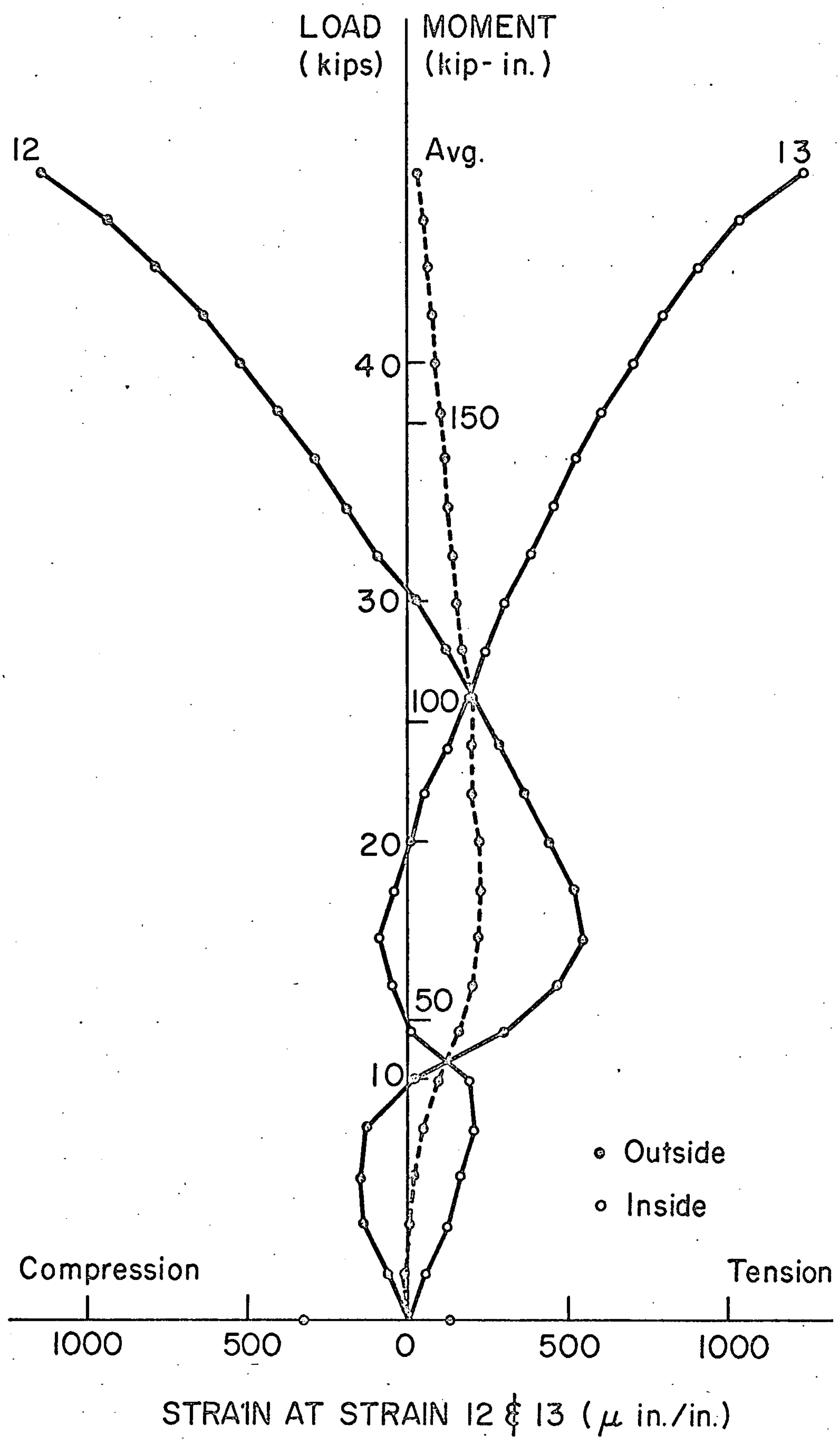


Fig. 15 VERTICAL WALL STRAINS OF CONNECTION WALL ABOVE CONNECTION OF 8-1

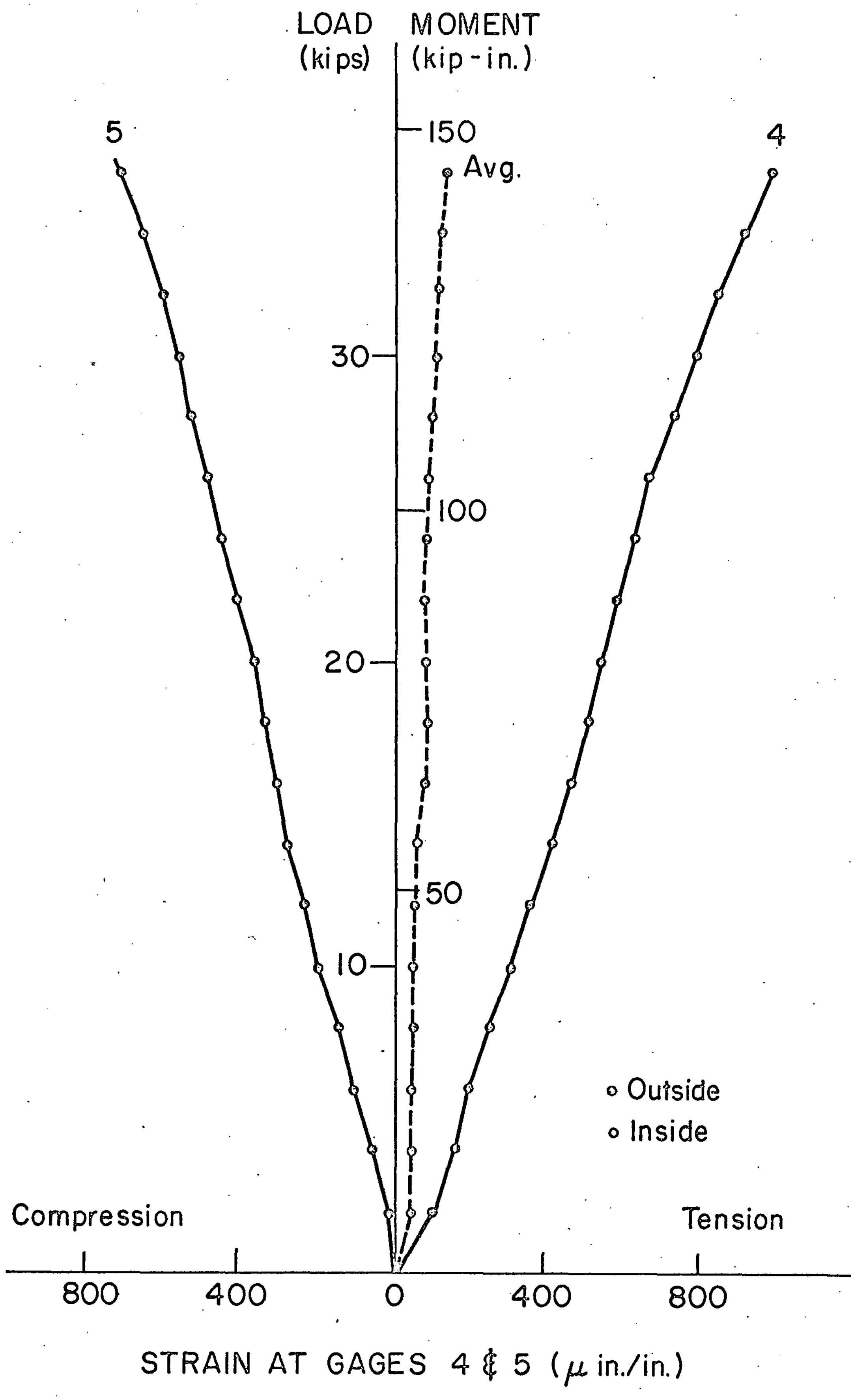


Fig. 16 VERTICAL WALL STRAINS OF UNLOADED WALL ABOVE CONNECTION OF 8-2

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